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Reliability Assessment of a Bridge with an Orthotropic Deck Subjected to Extreme Traffic Events

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ABSTRACT

When dealing with the construction of a bridge or the assessment of an existing bridge to traffic loads, one important point is the prediction of reliability levels for critical details to the expected traffic loads in its remaining lifetime: this is done here for details of a steel-orthotropic bridge deck based on limited traffic monitoring data. A comparison of results from different statistical approaches is made by analyzing the recorded data for the traffic actions: to do that, the work begins with the writing of limit state functions for the ultimate limit state using various probability distributions, to evaluate the corresponding reliability indexes. Indeed, three methods to assess extreme values, Generalized Extreme Value, Peaks-over-Threshold and Level Crossing Counting, are applied. Therefore, one of the extrapolation methods that have been used in the background works for the European Norms (Eurocode 1) is treated here. Moreover, the comparison with the European design load model and the corresponding ultimate limit state is made.

1. Introduction

Several long-span bridges with steel orthotropic decks are currently under re-assessment in Europe, as they are exposed to increased traffic loads to which they are sensible. Moreover, to

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evaluate if such a structure operates safely, it is possible to use measured data, both for the actions (traffic, climatic, …) and for the structure itself. Using extrapolation methods, the reliability analysis for a reference period covering the entire operational life of the critical details of a structure can be achieved.

Several methodologies for reliability analysis exist currently, each of them adapted for different domains of application and different shapes of limit state functions. Therefore, choosing correctly the most proper reliability approach is related to the studied object, initial conditions, input data, and computational efforts [1]. This kind of approach is being used for various domains of expertise [2,3]: As bridge engineering point of the view, First and Second-Order Reliability Methods (FORM and SORM) is the most frequently used reliability method [4]. Moreover, it has been shown recently [5,6] not merely fatigue limit state, but also extreme values, serviceability, and durability have to be investigated for the bridges the reliability computation.

This work will compare the assessed reliability levels of a given structure at the end of its design life, using Limit State Functions (LSF) whose variables are assessed using various extrapolation methods. For bridges with a steel orthotropic deck, as studied here, details of the deck may be more critical than structural elements [7], which is why the current research is designated for details of the orthotropic deck of the Millau bridge: these details are the welds at the junction deck/longitudinal stiffener which are under the path of the wheels of the trucks. Their equivalent stresses are calculated by using a finite element model and applying the traffic loads corresponding to a 180 days Weigh-in-Motion (BWIM) data on a bridge, and then they are extrapolated to their characteristic values by using several extrapolation methods: Block Maxima (BM), Peaks-over-Threshold (POT) and Level Crossing Counting (LCC).

2. Methodology

To assess the reliability of detail, the LSF has to be written and assessed. Generally, an LSF is expressed as:

$$G(x) = R(x) - L(x) > 0, \tag{1}$$

where $L(x)$ denotes the probabilistic model of a load effect – equivalent stress in the critical welding between a rib and the deck plate – and $R(x)$ represnts the probabilistic model for the welding resistance. In this work, as we will focus on the modeling of the loading, we will consider the resistance as remaining constant and equal to with initial resistance, so $R(x) = f(R_0)$ with R_0 the initial resistance of the material. Here R_0 indicates by the ultimate strength of steel, described using a lognormal distribution associated with the assumed covariance $CoV = 0.05$. The probabilistic model of the load effect $L(x)$ will be based on the calculed load effects of the recorded traffic data and extrapolated in time by one of the various approaches of the Extreme Value Theory (EVT).

Therefore, the reliability index β is computed for the given probability of failure P_f (as shown in Eq. (2)) concerning the probability density function $f_x(x)$ of the load distribution x and the probability cumulative distribution function (CDF) of the standard normal distribution Φ^{-1} (see Eq. (3)):

$$\beta = -\Phi^{-1}(P_f) = -\Phi^{-1}\left(\int_{G(x)\leq 0} f_x(x)dx\right), \tag{2}$$

$$where\ \Phi(x) = \frac{1}{\sqrt{2\pi}}\int_{-\infty}^{x} e^{-x^2/2}dx. \tag{3}$$

2.1. Block maxima method

The BM approach established based on a limit theorem referring that the Generalized Extreme Value Distribution (GEVD) is an estimation to explain sample maxima for sample sizes. This holds for the status of the BM being independent examens with identical distributions [8]. A BM series is presented by splitting the time history X of load influnce into intervals of a well-chosen size and determining the maxima in each interval. Various modifications of the BM method are available to handle inhomogeneities or dependence of events in the data-set, targeting at the approximated given idealized conditions of independent and identically distributed data in a residual subset to be utilized for the model fit.

One possibility [9] is to fit the BM series to a composite distribution model that describes the load influence of bridge based on a mixture of various loading scenarios, denoting the various number of trucks on the bridge having a contribution to a maximal value of the LE. The model considers the real loading scenarios of the bridge. Hence it concludes to greater accuracy for extreme value [10]. Based on separate GEVD fits $G_j(z)$ with model parameters $[\mu_j, \sigma_j, \xi_j]$ to the corresponding BM series for each of the different loading event types, the distribution of load effects $G_C(z)$ resulting from the mixture of these loading scenarios asymptotically methodes the following composite distribution [11,12]:

$$G_C(z) = \prod_{j=1}^{N} G_j(z) = \exp\left\{-\sum_{j=1}^{N}\left[1 + \xi_j\left(\frac{z-\mu_j}{\sigma_j}\right)\right]^{-\frac{1}{\xi_j}}\right\}. \tag{4}$$

because of the small effect of the considered load influence in the current case study (see Section 3), it was detected that the main contribution for extreme values of LE results from events with one single vehicle. Hence, there is no need for further classification for this study. The LSF (1) based on the BM takes the following form (with a reference block size d_{ref} and return period d_{return}):

$$G_{bm} = R - S_{bm}^{return}(\mu_b, \sigma_b, \xi_b) = R - \begin{cases} \mu_b - \frac{\sigma_b}{\xi_b}\left[1 - (d_{ref}/d_{return})^{-\xi_b}\right], & \xi_b \neq 0, \\ \mu_b - \sigma_b \log(d_{ref}/d_{return}), & \xi_b = 0, \end{cases} \tag{5}$$

where $\mu_b \sim \mathcal{N}(\mu_{\mu_b}; s_{\mu_b})$, $\sigma_b \sim \mathcal{L}(\mu_{\sigma_b}; s_{\sigma_b})$, $\xi_b \sim \mathcal{N}(\mu_{\xi_b}; s_{\xi_b})$ are location, scale, and shape parameters of the fitted GEVD, and where $\mathcal{N}$ and $\mathcal{L}$ are respectively the Normal and the Log-Normal distributions.

The model is presented in Figure 1 for the block of a week (five working days). It should be stated that working days are only considered to ignore the effects of weekends & holidays, while a less number of the heavy vehicles transfer the bridge: these LEs are therefore not imperative when measuring the constraints of the LSF as they are less/no damaging, and these do not follow the same probability of distribution.

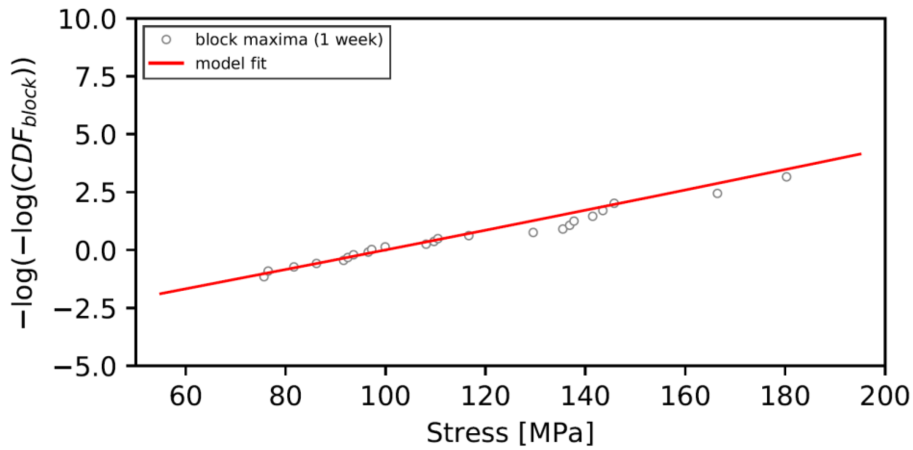

Fig. 1. Model fit, Block Maxima, 1 week.

2.2. Peaks-over-threshold approach

The POT method has been recently demonstrated an acceptable resolution to predict extreme traffic events [13,14]. As a time-series process, "peak" values of load effect (LE) that fall overhead a given threshold, are matched with the Generalized Pareto Distribution (GPD). It has been recently determined [15] that the efficiency of this method is rational concerning the assumption of the normal distribution of return level estimators. The conditional cumulative distribution function (conditional CDF) of Y given $X > u$, denoted by $F_u(y)$, can be expressed as:

$$F_u(y) = P[Y \leq y | X > u] = \frac{F(y+u) - F(u)}{1 - F(u)}, \tag{6}$$

in which $Y = X - u$, where $F(u)$ is the CDF of random variable X. The threshold overindulgences are shown by Y_i so that $Y_i = X_i - u$. The approach is effective only if $Y_j = X_j - u \geq 0$, $X_j \geq u$ for $\xi \geq 0$ and $u \leq Y_j \leq u - \sigma/\xi$ for $\xi < 0$. Moreover, the following conventions are made: (i) distinguished probability distribution of the random variables X_i, (ii) the random variables X_i are independent, (iii) the threshold u is adequate.

Based on the general principle of the POT method, which has been established a few decades earlier [16], the (CDF) of threshold exceedances inclines to the upper tail of a GPD, associated with the shape and scale parameters like ($\sigma > 0$ and ξ), see Figure 2 (left).

$$G(Y_j; \xi; \sigma; u) = \begin{cases} 1 - \left[1 + \xi \left(\frac{Y_j}{\sigma}\right)\right]^{-1/\xi}, & \xi \neq 0, \\ 1 - exp(-\frac{Y_j}{\sigma}), & \xi = 0. \end{cases} \tag{7}$$

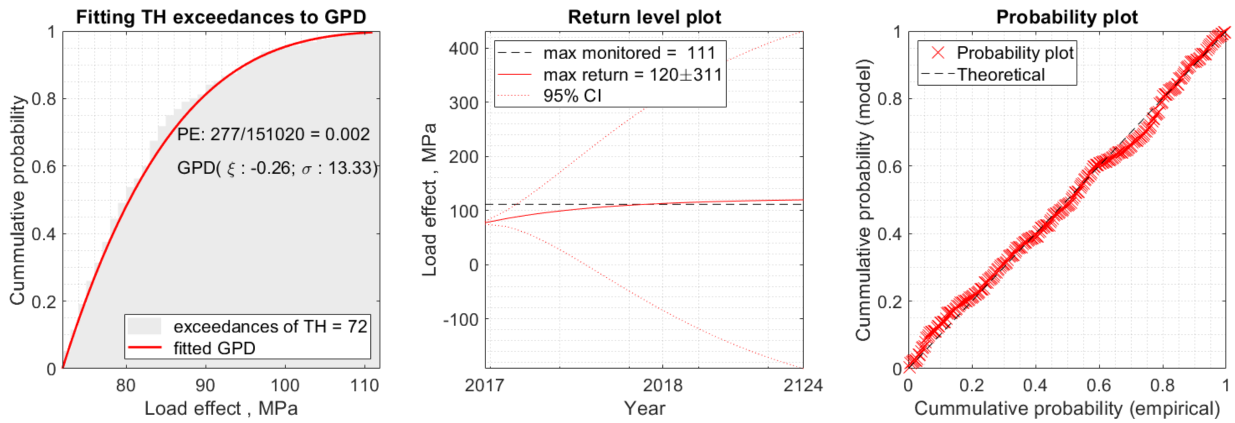

Fig. 2. Model fit, Peaks-over-threshold approach.

For a long period, estimates can be related to the CDF of extreme values associated with a shorter period [17]. In other words, the monitored values of a LE for a limited period can be used for extrapolation of the LE in time. The value of the p-observation return level $L_r(p)$ for the probability $P[X \leq X_i | X_i > u]$ with the Probability of Exceedance (PE) $\zeta_u = P\{X_i > u\}$, is a quantile that exceeds once every p observations, see Figure 1 (middle). Figure 1 (right) shows also the goodness of fit, where empirical values (crosses) should form a straight line to fit the theoretical model (dashed line). The procedure for selecting an optimized threshold has been described [18]. Therefore, for a fixed period p, the LSF (1) based on the GPD takes the following form:

$$G_{pot} = R - S_{pot}^{return}(p; u, \sigma, \xi, \zeta_u) = R - \begin{cases} u + \frac{\sigma}{\xi}[(p\zeta)^{\xi} - 1], & \xi \neq 0, \\ u + \sigma \log(p\zeta), & \xi = 0, \end{cases} \quad (8)$$

where $\zeta_u \sim \mathcal{B}(\mu_{\zeta_u}; s_{\zeta_u})$ is the number of load effects that surpass the threshold, over a total expanse of monitored events, $\sigma \sim \mathcal{L}(\mu_\sigma, s_\sigma)$ and $\xi \sim \mathcal{N}(\mu_\xi; s_\xi)$ are statsitical parameters of a fitted GPD (see Equation (7)), $\mathcal{B}$ is Binomial distribution.

2.3. Level crossing counting

Rice formula [19] explained the mean rate $\nu(x)$ of up-crossings for a given level x during the reference period d_{ref}. The LCC method, where Rice's formula is fitted to the upper tail of an out-crossing rate histogram (ORH) corresponds to the assumption of a stationary Gaussian process relating the time variations of load effects on bridges [20]. The number of times is taken at the positive values that are overlapped upwardly in a LE time-history.

Using the normalization of the obtained results from the crossing histogram for a give reference period d_{ref}, the ORH is computed, in lieu of each level the mean rate of its up-crossing $\nu(x)$ during d_{ref}. This mean rate as a function of load effect level can be defined by Rice formula using the parameters of model $[a_0 = ln(\nu_0) - m^2/2\sigma^2, a_1 = m/\sigma^2, a_2 = -1/2\sigma^2]$. It is fitted to the significant tail regions of the ORH, see Figure 3.

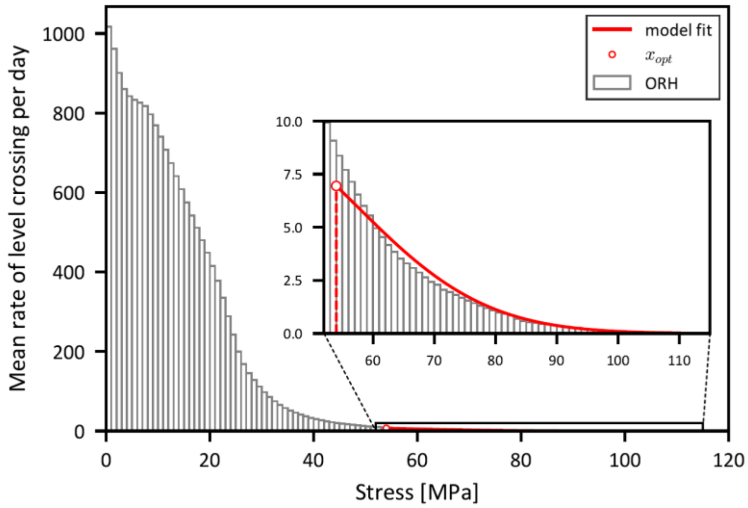

Fig. 3. Model fit, Level Crossing counting.

The proper selection of the initial point to fit with Rice's formula is central. It must be as low as feasible, to confirm the adequate representativeness for extrapolation of the data. Herein study, optimal initial points are recognized corresponding to the goodness-of-fit using a modified Kolmogorov test [20]. The LSF (1) ith consideration of the LCC is expressed as follows:

$$G_{lcc} = R - S_{lcc}^{return}(d_{return}, a_0, a_1, a_2) = -\frac{a_1}{2a_2} + \sqrt{-\frac{a_0}{a_2} + \left(\frac{a_1}{2a_2}\right)^2 - \frac{ln(d_{return}/T_{ref})}{a_2}}, \quad (9)$$

where d_{return} is the desired return period.

3. Application to the millau bridge

3.1. Instrumentation

Millau bridges is a cable-stayed bridge consisting of 8 spans with a total length of 2460 m. The stiff orthotropic steel deck is suspended by 11 steel cables for each span. Each span length is about 342 m. The cross-section of the deck is shown in Figure 4 [21], which illustrates that the bridge includes two lanes for both directions including slow-speed lane, where traffic is mainly dedicated to the heavy vehicles and fast speed lane.

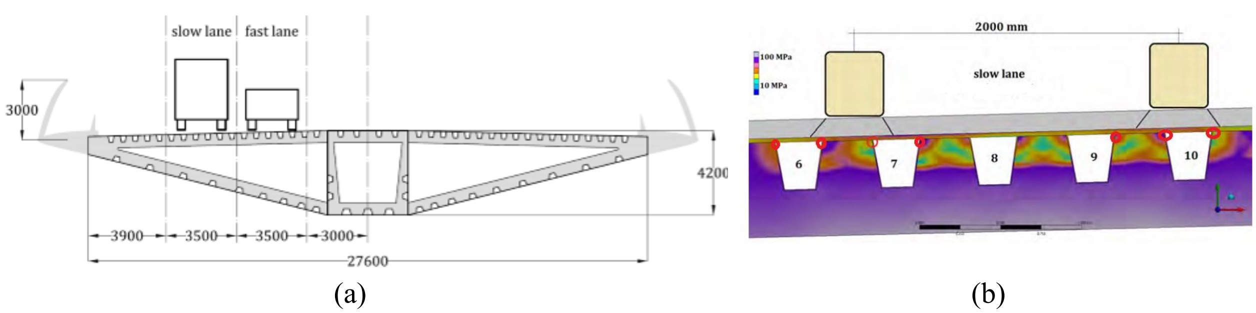

Fig. 4. Cross-section of the deck of the Millau Bridge: (a) Whole cross-section, and (b) focus on the considered welds under the wheel loads of the heavy trucks on the slow lane.

The data on traffic actions were provided from the BWIM system that was located in the middle of the first span of the bridge. It includes axle weights [kN] and spacing [m], vehicle speed [m/s], axles configuration, pavement temperature. Recordings were made between October 2016 and June 2017 with a total of 180 days of recorded traffic.

3.2. Effects of traffic actions

In this current study, the dangerous detail is selected concerning the stress connection on the welded area between a longitudinal stiffener and the plate of the orthotropic deck, which is located underneath of the truck wheels assuming that the vehicles are transferring from the middle of the slow lane. The stresses' results are obtained using the Finite Element Model (FEM) performed on the deck section. The model considers the self-weight of the bridge deck with its asphalt wearing cover, the type of passing vehicle, and amplitudes of axle loads from each axle are taken from reference [18], see Figure 5.

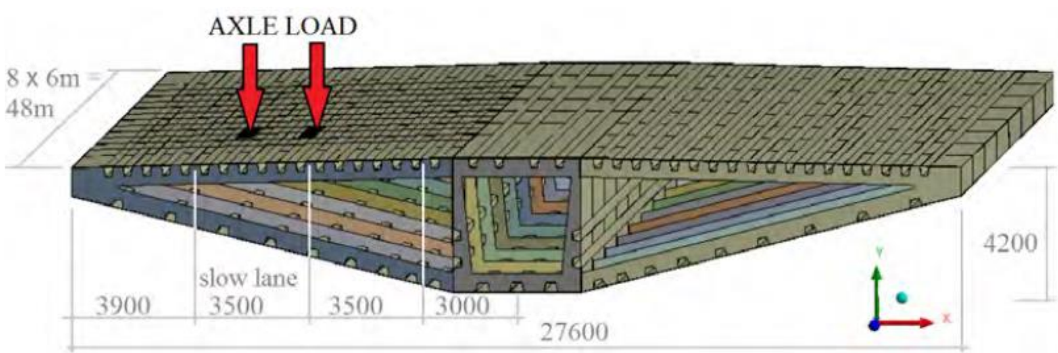

Fig. 5. Geometry for the FE model analyzed with ANSYS software.

The statistical parameters of the given random variables for each LSF corresponding to the explained methods are shown in Table 1.

Table 1
Random Variables for reliability modeling.

Case	Random variable		Distribution	Mean	CoV
Resistance	steel strength	F_u	Log-Normal	400 MPa	0.05
POT	threshold	u	Normal	102	0.05
	shape	ξ	Normal	-0.3	0.12
	scale	σ	Log-Normal	25.35	0.06
	PE	ζ_u	Binomial	8.2×10^{-4}	0.045
BM	location	μ	Normal	100.07	0.059
	scale	σ	Normal	23.57	0.198
	shape	ξ	Normal	-1.36×10^{-2}	17.503
* LCC	parameter	a_0	Normal	8.23	0.028
	parameter	a_1	Normal	-1.03×10^{-2}	0.798
	parameter	a_2	Normal	-8.06×10^{-4}	0.087

3.3. Comparison between various statistical approaches

To accomplish the reliability analysis, limit state functions are constructed for all described methods in Section 2. Accordingly, the UQLab software is utilized to compute reliability indexes using the SORM meth [22], according to the models' distributions for given load conditions explained in Section 3. The results of each method are tabulated in Table 2. The table represents the results for three-time references associated with one year, 50, and 120 years. The second column of Table 2 displays the calculated reliability index β_{pot} using POT approach, the third columsn shows the computed reliability index β_{bm} using on the BM method with considertion of th reference block corresponding to one week block (5 working days), and lastly, the fourth column of Table 2 denotes the obtained reliability index β_{lcc} using level up-crossings and Rice formula. The first three columns present the comparison of three extreme values methods for calculation of the reliability index concerning the load model. Also, the last column of Table 2 is related to the minimum values of β based on European standards [23].

As can be observed from this graph, two methods, POT and LCC, give similar curves, resulting from the smaller reliability indexes in comparison with the BM curve concerning a weekly block. This may construe that the BM approach, in this case, is pretty sensitive to the selected size of the block in addition to "extreme" data per block. Another point is that the application of the LCC methods, which is one of the approaches considered in the literature efforts of the European Code, which leads to the same reliability indices as taken from the POT method. Furthermore, it fits as well as the value calculated from the design load model LM1 for traffic load [24].

Table 2
Reliability index β for different studied cases.

β, reference	POT	BM	LCC	EN, LM1 [24]	β, EN [23]
1 year	9.8	19.7	9.6	-	4.7
50 years	7.6	12.5	8.0	8.0	3.8
120 years	7.4	11.4	7.7	-	-

4. Conclusions

This work was supported out based on the monitored traffic data given from the Millau bridge. Three extreme value methods were applied to measure the reliability of a susceptible detail, specifically the connection between a longitudinal stiffener and the steel plate of the orthotropic deck. The results of reliability are estimated for one year return period, for the 50-year as the reference period to relate with the European design models, at the end of the lifetime of the bridge.

Based on the obtained result of the performed analysis, the results of methods display that during the lifetime the reliability index of the most critical region of the deck is greater than the minimum required provision in the EN.

Also, the POT method and Rice formula demonstrated parallel outcomes that are closed enough to obtained results of the design load model (LM1). The reliability index obtained using the BM approach is far greater than the other methods, which stems from the size and variance of the selected block, which required further study. Therefore, it was concluded that the selection of a proper approach is related to the existing dataset. Hence, to enhance the confidence of the results, the reliability analysis should be elaborated using different approaches.

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